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BULLETIN NO. 27

TESTS OF BRICK COLUMNS AND TERRA COTTA BLOCK COLUMNS

BY

ARTHUR N. TALBOT

AND

DUFF A. ABRAMS

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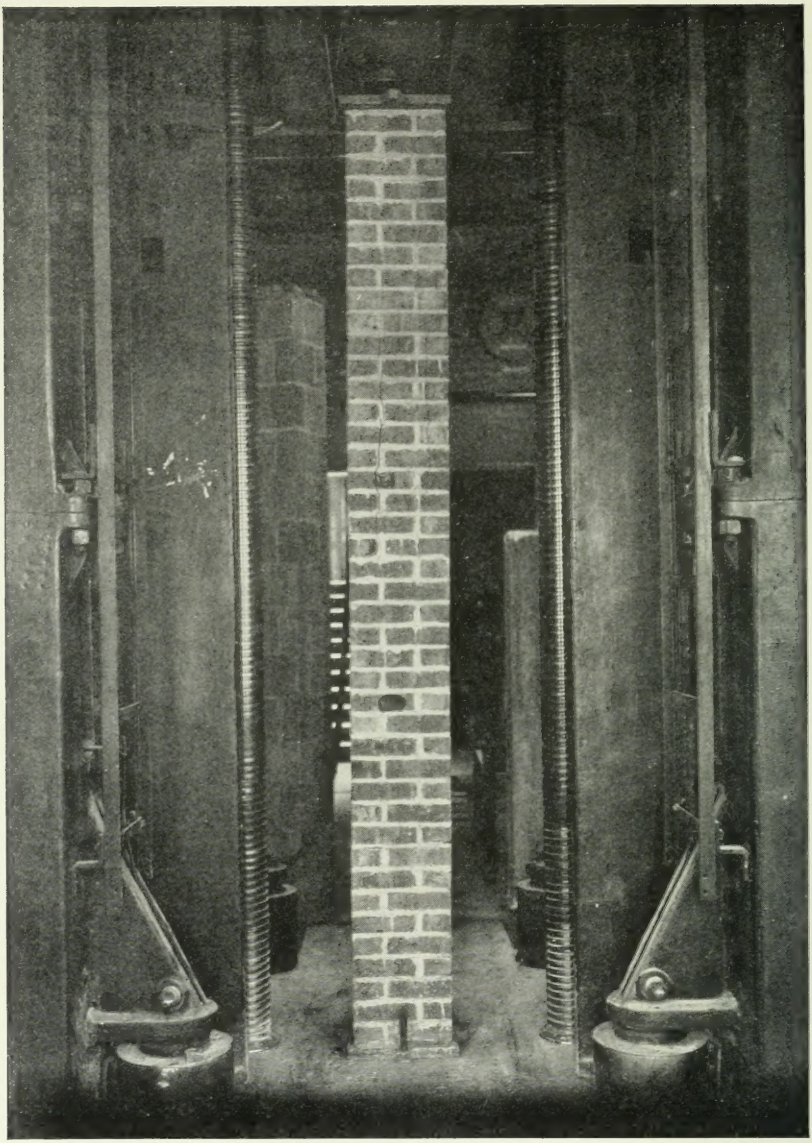

View of Brick Column No. 61 After Test, Showing Longitudinal Cracks.

UNIVERSITY OF ILLINOIS
ENGINEERING EXPERIMENT STATION

BULLETIN NO. 27 SEPTEMBER 1908

TESTS OF BRICK COLUMNS AND TERRA COTTA BLOCK COLUMNS

BY ARTHUR N. TALBOT, PROFESSOR OF MUNICIPAL AND SANITARY ENGINEERING AND IN CHARGE OF THEORETICAL AND APPLIED MECHANICS, AND DUFF A. ABRAMS, ASSOCIATE IN THEORETICAL AND APPLIED MECHANICS.

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I. Introduction

1. *Preliminary.*—For brick, stone and terra cotta masonry structures, designers commonly use stresses which are low in comparison with the strength of the constituent materials. Various reasons exist for this,—possible variations in material, chance for poor workmanship, opportunity for settlement and resulting unconsidered stresses, the desire to avoid tensile stresses in masonry, imperfect knowledge of the properties of the completed structure, etc. In some types of structure mass and volume are important considerations and strength is subordinate. In many applications, however, the strength of the structure is of first importance, and a knowledge of the actual properties of the structure is essential to safe and economical construction. Frequently designers use stresses as high as their information and the usual rules of practice will permit and make sections as small as these considerations warrant. The utilization of the working capacity of a structure is generally advantageous. This is especially true when the better grades of material are used and when the workmanship is known to be first-class. Much information is available on the strength and other properties of constituent materials like mortar, brick, stone, and terra cotta, but relatively little is known of the properties of built-up structures. Anything that adds to our knowledge of such construction should be helpful; and it was with a view of obtaining further information on the properties of short built-up columns or piers that the tests herein described were undertaken.

The work described in this bulletin includes tests of 16 short columns or piers of brick and 16 columns built of terra cotta blocks. Although long column action seems not to enter into these compression test pieces, they will be termed columns by analogy to the use of the term column for other compression members of no greater slenderness ratio. The length varied from 10 ft. to 12½ ft. In lateral dimensions the brick columns were 12½ x 12½ in., and in the terra cotta block columns the range was from 8½ x 8½ in. to 17½ x 17½ in. The brick used in the construction of the columns were of two grades,—an excellent class of building brick and a soft grade selected as representative of inferior brick. The terra cotta blocks were of good quality. Different qualities of mortar and different grades of workmanship were used. The loads were

applied continuously to failure in most cases, but tests were also made with repeated applications of the same load. Central loading of the column was used in most cases, but in some tests the load was applied eccentrically.

For convenience of reference in the consideration of the work not described in this bulletin, it has seemed well to mention tests made in other laboratories, and a brief summary of tests of brick piers is given in the following pages. The use of terra cotta blocks in compression members is not common, and there is little in print on the properties of this material.

2. *Watertown Arsenal Tests of Brick Columns.*—The Ordnance Department, United States Army, has from time to time made tests on brick piers at Watertown Arsenal. These tests are the principal source of data on strength of brick columns and the results are valuable. The tests are reported in several of the annual volumes of "Tests of Metals, Etc." Generally, only one test specimen was made for each variation in materials or form of structure. The exact conditions of the tests and the observed data are given in the reports of the tests. The following statement gives a summary of the number of tests and the nature of the investigations.

In "Tests of Metals, Etc." for 1884 are given the results of tests of 33 brick piers ranging from 8 x 8 in. to 16 x 16 in. in lateral dimensions and from 16 in. to 10 ft. in length. Lime mortar, natural cement mortar, and portland cement mortar were used. The results show clearly the increased strength obtained in columns built with the stronger mortar. The compressive strength of the columns ranged from 6 per cent to 16 per cent of the compressive strength of the individual brick. The tests show that columns having joints broken at every course have somewhat less strength than those with the joints broken every sixth course. The tests also show the effect which the load causing the first crack produces on the amount of compression and the amount of set in the column.

In "Tests of Metals, Etc." for 1886 are given the results of tests of 53 brick piers ranging from 8 x 8 in. to 16 x 16 in. in lateral dimensions and from 2 ft. to $12\frac{1}{2}$ ft. in length. Natural cement mortar (1-2) was used. The smaller the ratio of length to lateral dimension the greater was the compressive strength obtained. In cases where bond stones about 3 in. thick were used,

the strength obtained was greater than for a brick column of the same length but was less than for one whose height was equal to the distance between the bond stones. In the columns built with face brick, the strength developed varied from 18 per cent of the compressive strength of the individual brick in a column three diameters long to 12½ per cent in a column fifteen diameters long. In one column test reported in this volume no mortar was used. The column failed in the usual manner by the opening of longitudinal cracks at a load of 525 lb. per sq. in. or about 4 per cent of the strength of the brick. This test indicates the value of the mortar in distributing the load over the surface of the brick.

In "Tests of Metals, Etc.," for 1904, tests of 26 brick piers 12 x 12 in. x 8 ft. are reported. The variation was in the mortar used and in the age. Most of the tests were made at an age of about 6 months. Neat portland cement mortar, 1–2 and 1–3 portland cement mortar, and 1–3 lime mortar were used. The results show the increased strength given by strong mortar. One column laid up with neat cement mortar gave an ultimate strength of 4700 lb. per sq. in.; others with the same mortar but with poorer brick carried less than 2000 lb. per sq. in. The brick used in these tests varied in compressive strength from 4470 to 12 700 lb. per sq. in., based on the average of five samples tested flatwise. The ratios of the strengths of the piers to the compressive strength of the brick were as follows: Neat portland cement mortar, range 23 per cent to 43 per cent, average of 8 tests 30 per cent; 1–3 portland cement mortar, range 16 per cent to 35 per cent, average of 7 tests 24 per cent; 1–3 lime mortar, range 9 per cent to 18 per cent, average of 10 tests 13 per cent. In two tests the 1–3 mortar gave higher values than the neat cement mortar. Some of these test columns were exhibited at the Louisiana Purchase Exposition at St. Louis in 1904.

In "Tests of Metals, Etc." for 1905, tests of 13 clay brick columns and 1 sand-lime brick column are reported. The columns were 12 x 12 in. x 8 ft. high, with solid or hollow cores. The 14 piers were built from 9 different varieties of brick, laid in neat portland cement mortar, cement mortar, and lime mortar. They were tested at an age of about 5 months. The values ranged from 1500 lb. per sq. in. for the sand-lime brick to 4552 for clay brick, laid in neat portland cement mortar. Those laid up with 1–3 mortar gave values from 900 to 3422 lb. per sq. in. No tests of individual brick are given.

In "Tests of Metals, Etc." for 1906 are recorded tests of 15 brick piers 12 x 12 in. x 8 ft. Neat portland cement mortar, 1–3, 1–5, and 1–7 portland cement mortar, and 1–3 lime mortar, and 1 cement to 1 lime mortar were used. Tests were made at about 8 months. The results ranged from 853 to 3437 lb. per sq. in. for clay brick columns and 450 to 1400 lb. per sq. in. for sand-lime brick. In cases where the same brick were used nearly identical values were obtained by using 1–7 portland cement mortar and 1–1 lime mortar.

In "Tests of Metals, Etc." for 1907 are given tests of 32 columns, generally 11 x 11 in. or 12 x 12 in. in lateral dimensions and 8 ft. long. Neat portland cement mortar, 1–1 and 1–3 portland cement mortar, and lime mortar were used. The results show in a very interesting way the effect of the strength of the mortar used. The compressive strength varied from 730 to 5608 lb. per sq. in. The column which developed the highest compressive strength was built with paving brick. The columns built with portland cement mortar gave very high compressive strengths at early ages. One column, built of paving brick laid on edge in neat portland cement mortar, when 4 hours old held 2106 lb. per sq. in.; a similar column 2 days old carried 2733 lb. per sq. in.; and a similar one 7 days old 4514 lb. per sq. in. The columns tested at very early ages showed lack of stiffness and exhibited signs of distress at very low loads. In the column tested at 4 hours' age popping sounds were heard at 400 lb. per sq. in. and a crack appeared at 450 lb. per sq. in.—about 21 per cent of the maximum,—whereas in the columns tested at 2 and 7 days the first cracks appeared at 51 per cent of the maximum load and in a similar column tested at 4 months the first crack came at 61 per cent of the maximum load. Tests in this series indicate that columns in which the bricks are laid on edge or joints broken every third or sixth course give values 8 per cent to 10 per cent higher than columns laid with joints broken every course. Each of these modifications in the method of laying the brick seems to have its effect whether employed separately or in combination.

3. *Cornell University Tests of Brick Columns.*—Two series of tests of brick piers made at Cornell University have been reported. The first series made in 1897-8 consisted of 18 piers 13 x 13 in. in section varying from $2\frac{1}{2}$ to $7\frac{1}{2}$ ft. in length. Two varieties of brick were used. The piers were laid in 1-2 portland cement

mortar, joints broken at each course, and were tested at 4 or 14 months of age. The piers failed under loads of 635 to 1092 lb. per sq. in., corresponding to values 21 per cent to 28 per cent of crushing strength of corresponding brick. Failure generally followed the opening of longitudinal cracks through vertical joints. The variation in length did not affect the load-carrying capacity of the piers. These tests are reported in Transactions of the Association of Civil Engineers of Cornell University, Vol. VI, 1897-8.

In 1898-9 a second series of tests of 14 piers was made. The piers were uniform in size and material,—13 x 13 x 80 in., laid in 1–2 portland cement mortar. The variation was in the form of bonding devices which were placed in the horizontal joints of some of the piers. The brick used gave a crushing strength of 3525 lb. per sq. in. The principal result of these tests was to show the effect of iron or steel straps, netting or plates, embedded in the mortar of the horizontal joints. The maximum loads ranged from 22 per cent to 46 per cent of the crushing strength of the brick. Following is a summary of the results for different conditions in per cent of the crushing strength of the brick: 1–2 portland cement mortar only, (3 tests) 30 per cent; iron straps ($1\frac{1}{4}$ x $\frac{1}{8}$ in.) every fourth course, (2 tests) 24 per cent; iron straps every sixth course, (1 test) 22 per cent; iron straps every eighth course, (1 test) 24 per cent; wire netting ($\frac{1}{2}$ in. square mesh, 0.045 in. wire) every second course, (2 tests) 33 per cent; wire netting every course, (2 tests) 46 per cent; iron plate (0.026 in. thick) every fourth course, (4 tests) 28 per cent. These tests are reported in Transactions of the Association of Civil Engineers of Cornell University, Vol. VIII, 1899-1900.

4. *School of Practical Science Tests of Brick Columns.*—During the years of 1895 and 1896, 17 short brick piers were tested at the School of Practical Science, Toronto. These piers were 9 x 9 in. in section and ranged from 21 to 73 in. in height. Eleven grades of brick were used. 1–2 lime mortar and 1–3 cement mortar were used. The age at time of test was generally $2\frac{1}{2}$ months; one test was made at ten days. Careful records of strength of mortar and properties of brick were kept. Of the piers loaded to failure those built of lime mortar (11 tests) gave values averaging 16 per cent of the strength of the individual brick loaded flatwise, while those built of cement mortar (5 tests) gave values averaging 46 per cent of the strength of the brick. For further details

of these tests see Digest of Physical Tests, Vol. I, No. 3, July, 1896.

5. *Acknowledgment.*—The investigation was made at the instance of architects and builders who were desirous of finding the properties of columns built with material manufactured in this state. Mr. F. W. Butterworth of the Western Brick Company, of Danville, Illinois, furnished the building brick. The terra cotta blocks were supplied by the National Fire Proofing Co., of Chicago. The tests were made in the Laboratory of Applied Mechanics of the University of Illinois as a part of the regular work of the Engineering Experiment Station.

II. Materials, Test Pieces, and Methods of Testing.

6. *Brick.*—Two varieties of brick were used in these tests. The columns numbered 51 to 72 inclusive were built of high-grade, hard-burned, shale building brick, manufactured by the Western Brick Co., of Danville, Illinois. Columns No. 81 and 82 were built of soft, under-burned clay brick manufactured by the Sheldon Brick Co., of Champaign, Illinois, and selected from the soft brick of the kiln. The two varieties of brick represent extremes in quality. The soft brick was softer than would be used in building construction. This extreme was chosen better to bring out the effect of quality. The two varieties of brick will be referred to hereafter as "shale building brick" and "underburned clay brick" or "shale brick" and "clay brick". The results of compression tests of individual brick are given in Table 1, page 10. In all compression tests the specimen was first bedded in plaster of paris. In order to obtain an independent check on the quality of the brick used, transverse tests were made from which the modulus of rupture was computed. The results of these tests are given in Table 2, page 11. The tests of the brick in compression and cross-breaking gave results as uniform as are ordinarily found for this kind of material. The ratio of the modulus of rupture of brick tested edgewise, to the compressive strength of whole brick tested flatwise is 0.155 for the shale brick and 0.123 for the clay brick.

Fig. 1, page 9, gives the rate of absorption and the final amount of water absorbed by the two varieties of brick and by the terra cotta blocks. Each curve represents the average of 10

samples selected at random from the material furnished. The bricks and blocks were dried for about 48 hours on a steam radiator before the initial weight was taken. The same brick and blocks were afterward used in compression or transverse tests.

7. *Terra Cotta Blocks.*—The terra cotta blocks were furnished by the National Fire Proofing Co., of Chicago. The blocks came in two shipments. Columns No. 1 to 7, (tested January, 1907) were made from the first shipment, and the remainder of the terra cotta block columns from the second shipment. There was apparently no difference in the blocks in the two shipments, but both inspection and tests showed that the blocks of the second lot were more perfectly cut. In the first lot the ends or bearing faces were concave; in the second lot they were more nearly plane.

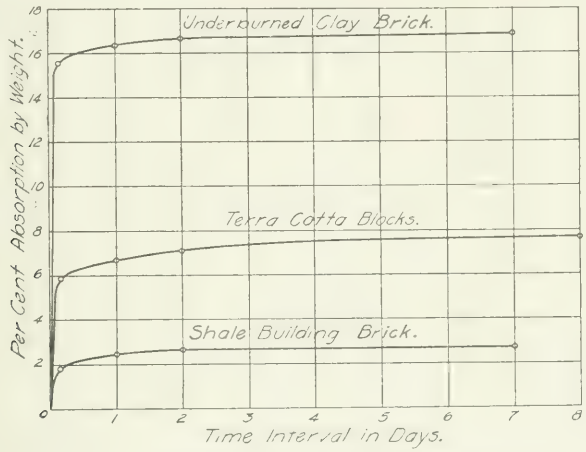


FIG. 1. ABSORPTION TESTS OF COLUMN MATERIALS.

Attention will be called later to the significance of this statement. It seems probable that the blocks represent a selected quality of the output of the factory. The size of the blocks was about 4 in. x $8\frac{1}{2}$ in. x 8 in. A small quantity in the second shipment were 4 in. x $8\frac{1}{2}$ in. x 4 in. The results of the tests of the small blocks in compression and in cross-breaking are included with the others, although none of them were used in building the columns. A sketch of a block with dimensions is given in Fig. 2, page 14. The gross section of each block was about 34 sq. in. and the net section about 28.75 sq. in., or 84.6 per cent of the gross section. The results of compression tests of individual blocks are given in Table 3, page 12.

TABLE 1.

Compression Tests of Brick.

Ref. No.	Dimensions, inches			Dry Weight pounds	Ultimate Load pounds	Compressive Strength lb. per sq. in.
	Width	Length	Depth			
Shale Building Brick						
1	3.95	8.25	2.35	6.36	364 000	11 150
2	3.90	8.20	2.42	6.48	312 000	9 750
3	3.92	8.25	2.40	6.42	259 000	8 020
4	3.85	8.25	2.35	6.46	223 000	7 030
5	3.95	8.15	2.32	6.00	320 000	9 950
6	4.00	8.32	2.35	6.28	365 000	11 000
7	3.85	8.25	2.30	6.01	320 000	10 090
8	3.85	8.20	2.30	6.10	349 000	11 050
9	3.95	8.20	2.33	6.11	264 000	8 150
10	3.95	8.25	2.30	6.56	462 000	14 150
11	4.00	8.34	2.30	6.29	457 000	13 700
12	3.92	8.30	2.37	6.50	348 000	10 700
13	3.90	8.25	2.40	6.51	359 000	11 150
14	3.90	8.20	2.30	6.51	453 000	14 150
15	3.95	8.30	2.40	6.44	378 000	11 520
16	4.05	8.35	2.25	6.36	373 000	11 020
17	4.00	8.30	2.40	6.40	324 000	9 750
18	3.90	8.28	2.43	6.49	324 000	10 040
Av.						10 690
Underburned Clay Brick						
1	4.00	8.25	2.40		134 300	4 070
2	4.05	8.35	2.40		92 800	2 750
3	4.15	8.30	2.43		99 200	2 870
4	4.00	8.00	2.45		110 400	3 450
5	3.75	7.60	2.45		238 900	8 390
6	4.05	8.25	2.40		73 200	2 190
7	4.05	8.30	2.30		108 300	3 230
8	4.00	8.00	2.40		107 900	3 370
9	4.00	8.20	2.45		159 000	4 850
10	4.07	8.40	2.55		83 200	2 430
12	4.10	8.10	2.45		175 700	5 290
13	4.15	8.25	2.40		95 500	2 790
14	4.10	8.20	2.35		119 600	3 560
15	4.00	8.00	2.30		181 500	5 670
16	4.00	8.00	2.40		125 900	3 930
Av.						3 920

All brick were bedded in plaster of paris before testing.
Seven shale brick tested on end averaged 7800 lb. per sq. in.
Seven clay brick tested on end averaged 3530 lb. per sq. in.

TABLE 2.

TRANSVERSE TESTS OF BRICK.

Ref. No.	Dimensions inches		Total Center Load pounds	Modulus of Rupture lb. per sq. in.	Remarks on Failure
	Depth	Width			
			Shale Building Brick		
1	3.96	2.38	4900	1180	Near middle.
2	3.86	2.39	6100	1540	do.
3	3.97	2.38	4100	963	do.
4	3.87	2.45	5600	1385	do.
5	3.85	2.37	5700	1450	At middle.
6	3.94	2.44	7600	1800	do.
7	3.95	2.48	8000	1860	do.
8	3.90	2.40	8000	1970	Near middle.
9	3.80	2.35	7300	1940	Diagonal break.
10	3.92	2.40	7300	1780	Angular break near middle.
Av.				1670	
			Underburned Clay Brick		
1	4.00	2.40	2000	516	Near middle.
2	4.08	2.48	1900	413	do.
3	4.00	2.48	2250	510	do.
4	4.05	2.45	1300	292	do.
5	3.95	2.46	2400	578	Near middle. Very regular.
6	4.00	2.45	2800	643	At middle.
7	4.05	2.40	2100	480	Near middle.
8	4.00	2.40	2000	469	do.
9	4.02	2.40	2100	489	At middle.
10	4.06	2.46	1900	423	do.
Av.				481	

Notes: The bricks were tested on edge, with a span of 6 in. The load was applied at the middle of the span, with a machine speed of 0.02 in. per minute. Curved pedestals, standing on a wooden block, were used. Metal bearing plates distributed the load across the surface of the brick at all points.

TABLE 3.

COMPRESSION TESTS OF TERRA COTTA BLOCKS.

Blocks loaded on end.

Ref. No.	Dimensions, inches		Dry Weight pounds	Total Load pounds	Compressive Strength lb. per sq. in.		Remarks
	Section	Height			on gross area	on net area	
Blocks as used in columns tested January, 1907							
1	4.10×8.50	8.00	17.12	99 000	2840	3350	Each end concave 1-16 in.
2	4.10×8.45	8.00	17.16	135 000	3900	4600	Each end concave $\frac{1}{8}$ in.
3	4.05×8.45	8.00	17.03	124 000	3630	4290	Ends concave $\frac{1}{8}$ to 1-16 in.
4	4.00×8.45	8.00		155 000	4580	5410	Ends fairly straight.
5	4.10×8.55	8.00	17.05	108 000	3080	3640	Ends concave 1-16 to $\frac{1}{8}$ in.
6	4.10×8.52	8.00	17.20	108 000	3090	3650	Each end concave $\frac{1}{8}$ in.
7	4.05×8.45	8.00	16.98	110 000	3220	3800	Ends concave 1-16 to $\frac{1}{8}$ in.
8	4.10×8.45	8.00	16.62	119 000	3440	4060	Ends straight.
Av.					3472	4100	
Blocks as used in columns tested January, 1908.							
1	3.95×8.25	8.00	15.82	194 000	5980	7100	One end concave 3-32 in.
2	3.92×8.23	7.92	15.65	232 000	7480	8590	Ends straight.
3	3.94×8.23	8.00	15.88	214 000	6550	7770	1-16 in. on one end, other straight.
4	3.96×8.30	8.00	15.92	251 000	7635	9070	One straight, one concave 1-16 in.
5	3.96×8.25	7.90	15.63	145 000	4440	5250	One end concave 1-16, one convex 1-16 in.
6	3.95×8.20	8.00	15.74	182 700	5640	6660	do.
7	4.00×8.25	8.00	.....	127 500	3860	4560	do.
8	3.90×8.20	7.95	... .	174 200	5450	6450	do.
9	4.00×8.35	8.00	.....	116 700	3490	4120	do.
10	3.90×8.30	8.00	16.07	156 000	4820	5700	One end straight, one end concave 1-16 in.
11	4.00×8.25	8.00	15.64	122 000	3700	4380	do. $\frac{1}{8}$ in.
12	4.00×8.30	8.00	15.92	102 000	3070	3630	do.
13	3.90×8.25	8.00	15.69	108 000	3350	3960	do.
14	3.90×8.20	8.00	15.88	205 000	6420	7560	do.
15	4.00×8.35	8.00	15.63	180 000	5390	6360	do. 3-32 in.
16	4.00×8.30	8.00	14.70	123 800	3720	4400	One concave $\frac{1}{8}$ in. twisted, one convex 1-16 in.
17	3.90×8.20	8.10	15.93	218 700	6850	8100	One straight, one concave $\frac{1}{8}$ in.
18	3.95×8.30	8.00	15.62	177 000	5400	6380	do.
Av.					5170	6115	
1	4.05×8.50	3.95	8.28	168 700	4900	5800	
2	4.05×8.50	4.05	8.30	162 400	4720	5580	
3	4.00×8.45	4.05	8.20	125 000	3700	4370	
4	4.05×8.50	4.05	8.28	136 300	3960	4680	
5	4.05×8.45	4.00	8.39	158 000	4620	5460	
6	4.00×8.40	4.00	8.21	169 400	5040	5950	
7	4.05×8.40	4.10	8.28	149 800	4400	5200	
Av.					4475	5290	

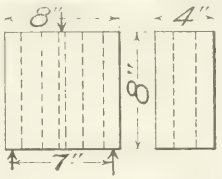
Notes: All blocks were bedded in plaster of paris before loading. The dimensions given are over all, and include two openings, 1½ in. x 1½ in., and one opening ⅞ in. x 1½ in. (See sketch of block, Fig. 2 page 14.) Tests were made on a 600 000-lb. testing machine; speed 0.05 in. per minute.

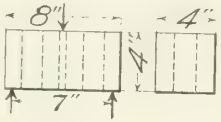
TABLE 4.

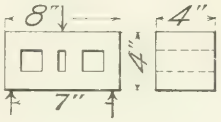
TRANSVERSE TESTS OF TERRA COTTA BLOCKS.

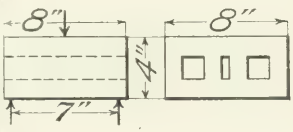
Blocks as used in columns tested January, 1908.

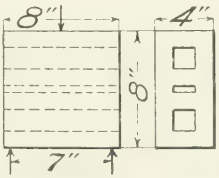
Ref. No.	Dimensions inches Width	Depth	Span in.	Total Center Load lb.	Modulus of Rupture lb. per sq. in.	Remarks	Method of Loading
1	4.0	8.0	8	12 700	970	Medium color	
2	3.95	8.0	8	11 300	870	do.	
3	4.1	8.0	8	14 000	1010	Very light inside	
4	4.0	8.0	8	11 700	880	Light inside	
5	4.0	8.0	8	16 300	1220	Medium color	
6	4.0	8.0	7	18 900	1240	Dark inside	
7	4.0	8.0	7	18 000	1180	do.	
Av.					1022		
1	4.0	4.0	8	2 500	750	....	
2	4.0	4.0	8	3 600	1080	....	
3	4.0	4.0	8	4 800	1440	....	
4	4.0	4.0	7	5 100	1340	Broke ½ in. from load	
5	4.0	4.0	7	3 200	841	Broke near middle	
6	4.0	4.0	7	3 700	974	do.	
7	4.0	4.0	7	4 900	1290	do.	
Av.					1102		
1	4.0	4.0	7	4 000	690	Broke through outer opening	
2	4.0	4.0	7	3 300	578	do.	
3	4.0	4.0	7	3 600	620	do.	
Av.					629		
1	8.25	4.0	7	11 200	912	Broke at middle. Light inside	
3	8.4	4.0	7	9 700	770	Break very regular	
3	8.35	4.0	7	8 800	708	do.	
Av.					797		
1	4.0	8.25	7	13 000	562	Diagonal failure	
2	4.0	8.25	7	16 300	707	do.	
Av.					635		











NOTES: In calculating the modulus of rupture the net section of the block was used, a deduction for the opening being made. The load was applied at the middle of the span in each case. Curved pedestals, standing on a wood block, were used. Metal bearing plates distributed the load across the block at all points. The pulling head of the machine moved at a rate 0.02 in. per minute in all these tests.

The crushing load on the block is given as computed for both gross and net areas. It will be noted that the condition of the ends recorded in the table has a decided influence on the strength of the block in this kind of a test. In the compression tests the terra cotta blocks began to spall at the corners at about 90 per cent of the ultimate load. As the ultimate load was approached, popping noises were heard and failure soon followed by the block being completely shattered by the formation of numerous longitudinal cracks. All the tests given were made at the same time under identical conditions. Results of transverse tests on terra cotta blocks are given in Table 4, page 13. These tests were made with blocks loaded on top, on side and on edge. The method of loading is indicated by sketches in the table.

Fig. 1, page 9, gives the rate of absorption as already described.

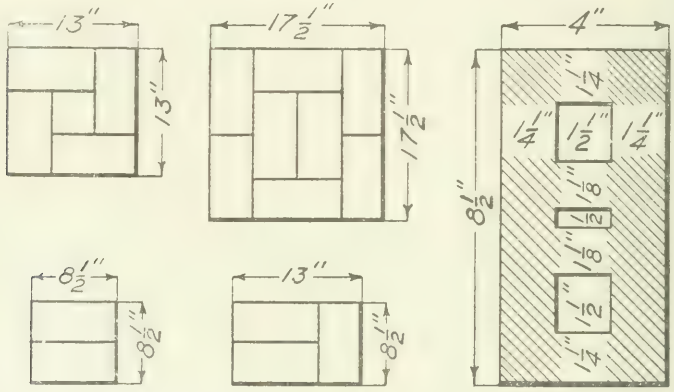


Fig. 2. Sketch of a Terra Cotta Block and Arrangement of Terra Cotta Blocks and Brick in Various Sections.

8. *Cement, Sand, and Mortar.*—Universal portland cement mortar was used in all 16 terra cotta block columns and in 13 of the brick columns. In two brick columns lime mortar was used and in one brick column Bricklayers' (natural) cement was used. The results of tensile tests of these cements are given in Table 5, page 15. The results of the test of the Universal cement for the two different years are given separately. The cement for the terra cotta block columns tested January, 1907 was from the 1907 lot; all other portland cement used was from the 1908 lot. The

sand was the same as was used for the reinforced concrete tests during the season of 1906-7. It came from near the Wabash river at Attica, Indiana. It was screened through a No. 10 sieve and contained a rather large amount of clay. Average values from mechanical analyses of the sand are given in Table 6, page 16.

TABLE 5.

TENSILE STRENGTH OF CEMENT.

Ref. No.	Ultimate Strength. lb. per sq. in.											
	1907 Universal (portland)				1908 Universal (portland)				Bricklayers' (natural)			
	Age 7 days		Age 28 days		Age 7 days		Age 28 days		Age 7 days		Age 28 days	
	Neat	1-3	Neat	1-3	Neat	1-3	Neat	1-3	Neat	1-3	Neat	1-3
1	410	187	680	370	563	244	764	319	120	40	175	87
2	470	200	670	330	809	248	885	336	120	38	210	87
3	360	120	560	360	728	232	776	285	100	40	130	81
4	405	145	570	290	699	242	754	292	115	35	140	84
5	320	195	600	295	702	229	763	315	100		190	87
Av.	379	171	617	326	700	239	788	309	111	38	169	85

The 1-3 briquettes were made from standard Ottawa sand.

All the values for each cement given in any horizontal line are from briquettes made from the same sample. Each value given for the Universal cement is the average of 5 briquettes.

The values given for the Bricklayers' cement are each from a single briquette test. All the tests are from a single sample of cement.

A 1-2 mortar was used in making 7 terra cotta block columns, a 1-3 mortar in 7 columns and a 1-5 mortar in 2 others. Of the brick columns, 11 were made with 1-3, and 2 with 1-5 portland cement mortar, 1 with 1-3 Bricklayers' (natural) cement mortar, and 2 with 1-2 lime mortar. The proportioning was by loose volume in all cases. The mortar was mixed in small batches as needed. The mixing was done by a bricklayer's helper, who used the ordinary methods for such work. In most cases as the columns were laid up, one or two 6-in. cubes were made from each batch of cement mortar and afterwards tested as a check on the quality of the mortar. The results of these tests may be found in Table 9, page 26, and Table 13, page 42. The value given for each column is the average of the tests of the cubes made from the mortar used in the corresponding column.

9. *Dimensions of Columns.*—The first 7 terra cotta block columns (those tested January, 1907) were built from 11 ft. 10 in. to 12 ft. 6 in. high and of four different cross sections, $8\frac{1}{2}$ in. x $8\frac{1}{2}$ in., $8\frac{1}{2}$ in. x 13 in., 13 in. x 13 in., and $17\frac{1}{2}$ in. x $17\frac{1}{2}$ in. The remainder of the columns, both terra cotta block and brick, were built about 10 ft. high and with a cross section about $12\frac{1}{2}$ in. x $12\frac{1}{2}$ in. The exact dimensions of the columns are given in Tables 7 and 11, pages 24 and 39. The arrangement of the blocks in the different sections is indicated in Fig. 2, page 14.

TABLE 6.

MECHANICAL ANALYSIS OF SAND.

Average of 5 Samples.

Sieve No.	Separation Size inches	Per cent Passing
10	.091	100
12	.067	87.3
16	——	80.0
18	.043	65.0
30	.027	42.7
40	.019	28.5
50	.013	17.3
74	.009	8.3
150	——	2.3
200	——	1.6

10. *Fabrication and Storage of the Columns.*—The columns were made in the Laboratory of Applied Mechanics of the University of Illinois. The work was done by experienced bricklayers. Columns No. 1 to No. 7, inclusive, were laid up by a workman sent for that purpose by the National Fire Proofing Co., of Chicago, the company which furnished the terra cotta blocks for these tests. This workman had had considerable experience in this particular kind of work and had previously built a number of columns for testing purposes. The remainder of the terra cotta block and all of the brick columns were built by a workman furnished by a local contractor, who was requested to select a representative bricklayer. He had had no previous experience in laying terra cotta blocks, but was thrown upon his own resources after receiving necessary instructions regarding dimensions, etc. The work of the latter man seems to be more nearly representative of what would be obtained in an ordinary case, although no

appreciable difference in the results can be traced to the difference in workmanship. No special care was taken in selecting the brick or blocks in building the columns except that from the second shipment of terra cotta blocks enough under-burned or apparently inferior blocks were set aside to build one column, (No. 17).

A base plate of cast iron about 1¼ in. in thickness, planed on each side, was leveled on the concrete floor and the column built upon it. The terra cotta blocks and clay brick were plunged into a bucket of water a few minutes before laying. The shale brick were so impervious that this was not necessary. All columns described herein were laid up solid, i.e., no hollow spaces were left, except the vertical openings in the terra cotta blocks. No attempt was made to fill these openings with mortar. Joints were broken in every course in all columns built. The brick were laid flatwise as in practice. The number of courses and the thickness of the mortar joints are given in the tables for each column. A cast-iron bearing plate similar to the one mentioned above was bedded in mortar on the top of each column. These plates were useful when moving the columns to the testing machine, and they served as bearing plates in the test. The columns composing a set (similarly built and tested) were built on different days in order that average conditions might govern and thus eliminate as far as may be the accidental effect of the conditions of workmanship and materials.

All of the columns which are designated in the tables as well laid were built with the care usually given to first-class masonry. In building five of the columns (brick columns No. 55 and 56, and terra cotta block columns No. 12, 13, and 14) designated in the tables as poorly laid, the workman was told to use less care in making horizontal joints and in filling the vertical joints. It was the intention to produce the effect of a hurried or careless workman. These columns were built in about three-fourths the time required for a similar one well laid.

Each column was left standing in the place in which it was built until time of test. The columns were kept in an upright position as they were transported to the testing machine by means of an overhead crane. No attempt was made to prevent the mortar in the joints from drying out. The temperature of the room in which the columns were made and stored varied from 55° to 70° F.

The columns were tested at an age of about 60 days. Two brick columns were tested at an age of 6 months. The exact age at time of test is given for each column.

11. *Methods of Testing.*—Fourteen terra cotta block columns were tested with concentric loading and two with eccentric loading; on some of the columns the load was applied in more than one way. One of the terra cotta block columns (No. 15) did not fail after several repetitions of the concentric load up to the capacity of the machine and was afterward loaded eccentrically until failure occurred. Terra cotta block columns No. 7 and No. 8 did not fail after receiving several applications of the full load of the testing machine. No. 7 was loaded eccentrically also but not to failure. They were set aside for further test at some future date. Two of the terra cotta block columns, No. 6 and No. 7, had a gradually increasing load applied and released in succession. Of the brick columns, 14 were tested with concentric load and 2 with the load applied eccentrically. In the test of No. 60 (shale brick, 1–3 mortar, age at test 6 months) failure was finally produced by repeating the maximum load a number of times.

In several cases when failure occurred at or near the top of the column enough of the top was removed so that the remaining portion contained no cracks, a bearing plate was placed in mortar on the top, and the resulting short column afterward tested. The results of these tests do not differ greatly from those of the original columns. The slight increase in compressive strength may be accredited to the increase in the strength of the mortar during the interval.

All the tests were made in the 600 000-lb. Riehle vertical screw testing machine in the Laboratory of Applied Mechanics of the University of Illinois. The machine head moved at the rate of 0.05 inches per minute; except in some cases, where the load was repeated, when a speed of 0.10 inches per minute was used. The load was generally applied by increments of about 25 000 lb. on the column, but for the columns built of lean mortar or of clay brick the increment was smaller. The load was applied through a spherical bearing block, except where the column was loaded eccentrically. In arranging the columns for eccentric loading, a $\frac{1}{2}$-inch square steel bar about twenty inches long was placed one inch off the center of the column under the lower bearing block, and a similar bar in a corresponding position on top of the upper

bearing block. The load was applied to the column through these bars.

The measurements of longitudinal deformations were made by instruments which are shown in the views in Fig. 8. The yokes were so placed on the column that measurements were taken over a length of about 100 inches. On each side of the lower yoke an instrument was placed which read direct on the dial to thousandths and by estimation to ten-thousandths of an inch. After each increment of load was applied, the machine was stopped and readings of the extensometers taken before again increasing the load. Readings were generally taken at increments of load of 25 000 lb., although smaller increments were used in the tests of the columns in which clay brick or a lean mortar was used.

In some of the earlier tests the lateral deflection of the column from its original vertical position was measured by means of two fine silk threads and mirror scales placed on two faces near the middle of the column. The threads were fastened at points near the ends of the column and held taut. The amount of the center deflection would be indicated by the movement of the thread over the scale. The actual amount and direction of the deflection could be calculated by considering the scale readings as the rectangular components of the movement. The measurement of lateral deflection was finally discontinued, as the total amount of the movement was so small as to be insignificant. The total center deflection of a brick or terra cotta block column $12\frac{1}{2}$ x $12\frac{1}{2}$ in. in section, and 10 ft. long, in which the stronger mortar was used, was no greater than 0.03 in. to 0.08 in.

III. Experimental Data and Discussion.

A. Brick Column Tests.

12. *Brick Column Test Data.*—Fourteen columns of shale building brick and two columns of underburned clay brick, all approximately $12\frac{1}{2}$ x $12\frac{1}{2}$ in. in cross section and 10 ft. long, were tested. The make-up of the columns, method of applying the load and the age at test varied. Table 7, page 24, gives data on the dimensions and make-up of the brick columns. Table 8, page 25, gives data on the tests of the columns.

13. *Phenomena of the Tests of Brick Columns.*—As already stated, the brick columns were made in sets of two or three,

which were constructed and loaded similarly. With the exception of No. 60, these columns were loaded continuously to failure. In the test of No. 60, a load above the rated capacity of the machine was released and re-applied seven times before failure finally occurred. The phenomena of the continuous-loading tests will be described here, and the repeated-load test will be discussed in a succeeding paragraph.

In all of the brick column tests, the first evidence of distress observed was a faint popping sound which seemed to proceed from the interior of the column. The load at which this popping was first heard, expressed as a proportional part of the maximum load carried by the column, is given in the last column of Table 8, page 25. As the load was increased, the popping noises were heard more frequently and were louder. As the load was further increased, the action of the columns depended to a great extent on the quality of the mortar used. The columns in which the richer and stronger mortar was used gave little or no additional evidence of distress until a load a little below the maximum was reached, when spalling of the mortar at the corners of the column, or the formation of longitudinal cracks through the vertical joints gave warning of impending failure; after the beginning of spalling or the formation of longitudinal cracks was observed, the failure was generally very sudden and complete. In this class of columns (those with 1–3 portland cement mortar) the debris showed that failure was precipitated by the formation of longitudinal cracks through each vertical joint. These cracks generally extended throughout about the upper two-thirds of the length of the column, beginning near the middle and extending both ways. Such failures were extremely violent and sometimes involved hazard to observers and often proved destructive to measuring instruments. The failures came with such slight warning that, despite repeated efforts, the operator was unable, except in two tests (Columns No. 55 and 56), to stop the movement of the testing machine after evidences of failure were observed in time to avoid reducing the upper two-thirds of the column to a mass of broken bricks and mortar scattered over the Laboratory. The design of this machine is such that it may be stopped and the motion completely reversed almost instantly. It will be found by reference to the table that the columns referred to were the ones laid up carelessly. They were less rigid than the others of the same materials and hence took on load more

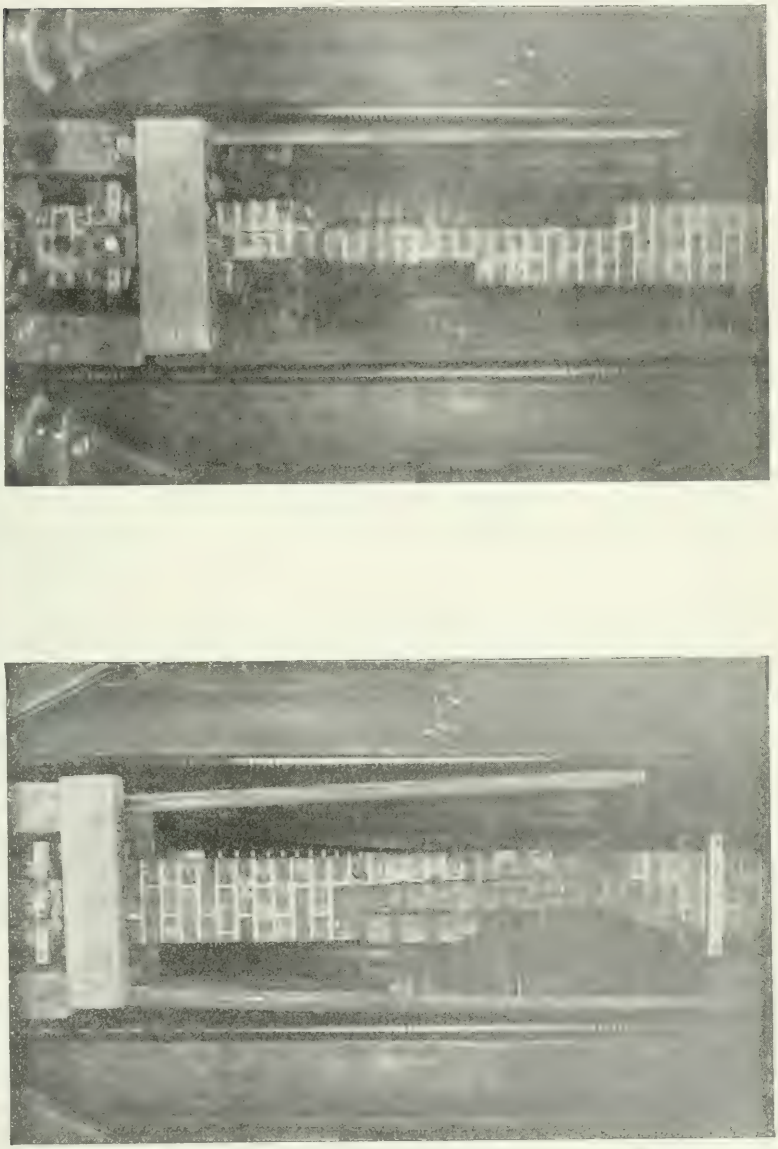


FIG. 3 VIEWS OF BRICK COLUMNS NO. 55 AND 56 AFTER TESTS SHOWING MANNER OF FAILURE.

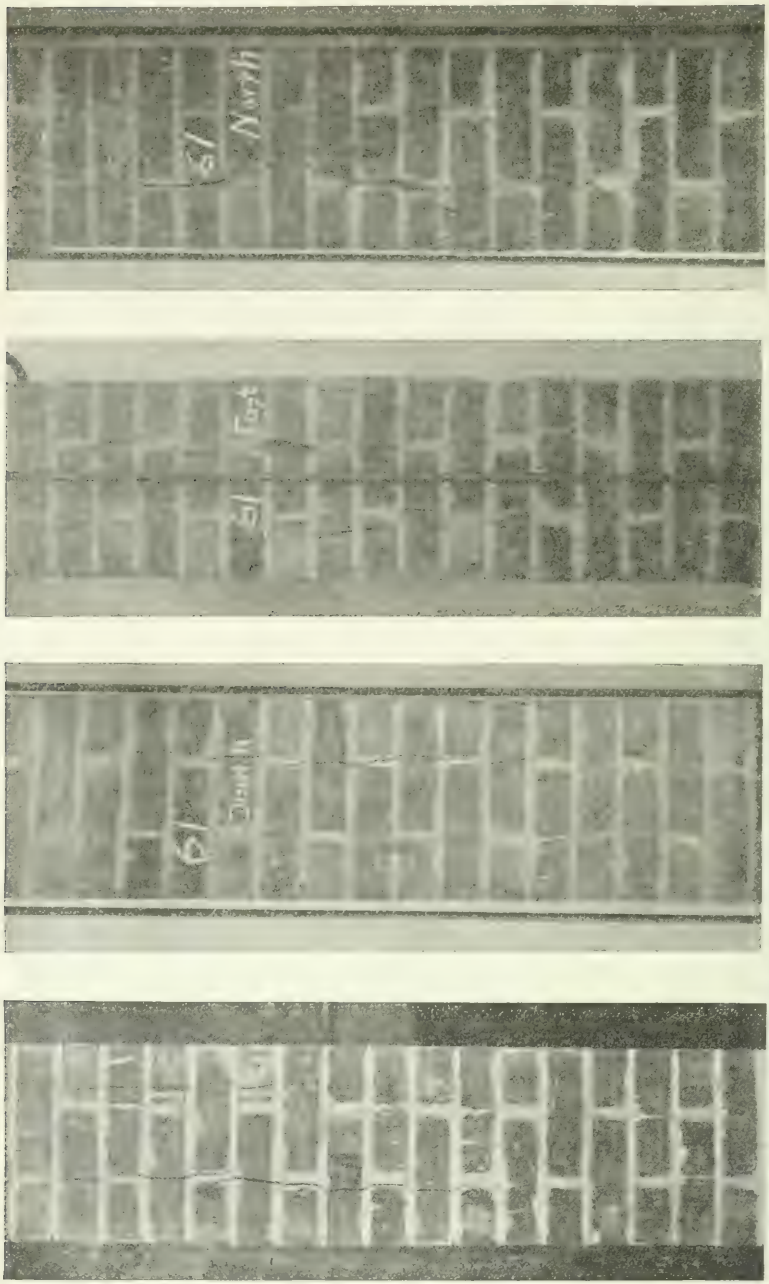


FIG. 4. VIEWS OF BRICK COLUMN NO. 61 AFTER TEST SHOWING CRACKS FORMED ON EACH OF THE FOUR FACES.

slowly. The photographs, Fig. 3, page 21, show the two brick columns (No. 55 and No. 56) in which the collapse was not complete; the smallest section of No. 55, about 2½ ft. below the top, is scarcely larger than one brick.

The phenomena of the test of the clay-brick columns did not differ greatly from those of shale-brick with 1–3 portland cement mortar, except that on account of the reduced rigidity the load was applied more slowly and the failures were less sudden and violent.

The failures of the columns in which 1–5 portland cement, natural cement, or lime mortar was used were similar to those described above, except that they rarely got beyond the spalling and cracking stage. In these tests the formation of the vertical cracks could readily be observed. The freedom from sudden collapse was probably due to the yielding of the joints and the fact that the testing machine does not follow such yielding instantaneously. It must not be inferred that under an actual load such columns would not have failed as suddenly and completely as the others. Where natural cement or lime mortar was used, the mortar gradually disintegrated and reduced to powder.

The photographs reproduced in Fig. 4, page 22, show the size and location of these vertical cracks in a brick column in which 1–5 portland cement mortar was used. Each view shows the condition of the upper half of one face of the column after failure. Some of the cracks could be traced nearly to the bottom of the column. This distribution of cracks is characteristic of the columns with stronger mortar, but, as stated above, it was impossible to discontinue the tests of the stronger columns at this stage. In these tests also the cracks and excessive spalling were generally confined to the upper two-thirds of the length of the column.

This phenomenon may be accounted for by the consideration that the mortar in the lower portion of the column is appreciably stronger than that above, due to the weight of the column coming upon it during the period of setting. This phenomenon has been observed in other tests designed for that purpose, both in this Laboratory and elsewhere. Recently two sets of 6-in. cubes (3 cubes in a set) were made from the same batch of concrete. Two cubes from each set were allowed to harden in the open air as usual, while the third was loaded with a pressure of about 10 lb.

per sq. in. The cubes were tested in compression after a period of 10 days; the cubes which set under pressure giving in each case a value of about 33 per cent in excess of the average of the other two.

The columns loaded eccentrically failed by the formation of vertical cracks parallel to the loading plane. They were thrown from the machine toward the side opposite the load.

TABLE 7.

DATA OF BRICK COLUMNS.

Col. No.	Characteristics of Column and Loading	Kind of Mortar	Length ft.-in.	Nominal Section in. x in.	Gross Area sq. in.	Number of Courses	Average Thickness of Joint inches
		Shale Building Brick					
51	Well laid, concentric load.	1-3 port. cement	9-10	12½ x 12½	157.5	43	.33
52	do.	do.	9-11⅓	12½ x 12½	158.8	43	.37
53	do.	do.	9-9	12½ x 12½	156.3	42	.38
59	do.	do.	9-11	12½ x 12½	158.8	43	.36
60	do.	do.	9-11	12½ x 12½	158.1	43	.36
55	Poorly laid, concentric load.	do.	9-11	12½ x 12½	158.1	43	.36
56	do.	do.	10-0	12½ x 12½	155.0	43	.38
57	Well laid, eccentric load. ($\epsilon = 1$ in.)	do.	9-10	12½ x 12½	156.9	43	.33
58	do.	do.	9-11¾	12½ x 12½	156.9	43	.38
61	Well laid, concentric load.	1-5 port. cement	9-9¾	12½ x 12½	159.4	42	.39
62	do.	do.	9-10	12½ x 12½	158.1	42	.40
91	do.	1-3 natural cement	10-0	12½ x 12½	158.8	43	.38
71	do.	1-2 lime	9-10	12½ x 12½	158.8	43	.33
72	do.	do.	9-8½	12½ x 12½	160.7	43	.30
		Underburned Clay Brick					
81	Well laid, concentric load.	1-3 port. cement	10-0½	12½ x 12½	161.3	40	.46
82	do.	do.	9-11½	12½ x 12½	161.9	40	.44

TABLE 8.

DATA OF TESTS OF BRICK COLUMNS.

Col. No.	Maximum Load		Age at Test days	Manner of Failure	Initial Modulus of Elasticity, lb. per sq. in.	Maximum Unit Shortening	Proportional Load at which Popping was First Noted
	Total pounds	Unit, lb. per sq. in.					
Shale Building Brick							
51	507 000	3220	66	Split along joint, top to bottom.	4 350 000	.00104	.59
52	557 000	3510	68	Total collapse with little warning.	5 450 000	.00110	.60
53	527 000	3370	65	do.	4 550 000	.00122	.50
59	601 000	3790	181	do.	4 700 000	.00106	.75
60	651 000	4110	181	Upper half shattered on seventh repetition of load.	5 350 000	.00129	...
55	454 000	2860	69	Total collapse with little warning.	3 550 000	.00102	.66
56	462 000	2980	68	Middle portion thrown out.	3 500 000	.00094	.65
57	427 000	2720	69	Split through joints, near top.	4 100 000	.....	.71
58	452 000	2880	67	Split through joints. Center thrown from machine.	4 700 000	.....	.52
61	350 000	2190	66	Cracked from bottom to top.	3 500 000	.00109	.57
62	358 000	2260	65	Total collapse after longitudinal cracks in top	3 000 000	.00129	.46
91	277 000	1750	66	Spalled at joints. Did not collapse.	800 000	.0027	.40
71	216 000	1360	67	Badly spalled. Mortar reduced to powder. Did not collapse.	101 000	.....	.38
72	248 000	1540	66	Column deflected noticeably to north, spalling and crushing on south. Did not collapse.	107 000	.....	.48
Underburned Clay Brick							
81	166 000	1030	63	Total collapse.	435 000	.00270	.91
82	177 000	1090	62	do.	430 000	.00250	.62

14. *Strength of Brick Columns.*—The results of the tests of brick columns, given in Table 8, page 25, show greater uniformity than has usually been found for test specimens of this kind. A summarized statement of average values from the tests is given in Table 9, page 26. The average value of the crushing strength of brick loaded flatwise was used in computing the ratio in the fourth column of the table. The average value of the unit-loads taken by the three shale brick columns laid up with 1–3 portland cement mortar and tested at 67 days, is used in computing the ratios in the fifth column of the table.

The columns which were built of shale brick, using 1–3 and 1–5 portland cement mortar, and loaded in a similar manner, give

TABLE 9.

SUMMARY OF TESTS OF BRICK COLUMNS.

Average Values.

Ref.	Characteristics of Columns	Average Unit Load, lb. per sq. in.	Ratio of Strength of Column to Strength of Brick	Ratio of Strength of Column to Strength of "A"	Crushing Strength of 6-in. Mortar Cubes, lb. per sq. in.	Ratio of Strength of Column to Strength of Cubes
	Shale Building Brick					
A	Well laid, 1–3 portland cement mortar, 67 days.	3365	.31	1.00	2870*	1.17
B	Well laid, 1–3 portland cement mortar, 6 months.	3950	.37	1.18	.....	.....
C	Well laid, 1–3 portland cement mortar, eccentrically loaded, 68 days.	2800	.26	.83	.....	.....
D	Poorly laid, 1–3 portland cement mortar, 67 days.	2920	.27	.87	2870*	1.05
E	Well laid, 1–5 portland cement mortar, 65 days.	2225	.21	.66	1710	1.30
F	Well laid, 1–3 natural cement mortar, 67 days.	1750	.16	.52	305	5.75
G	Well laid, 1–2 lime mortar, 66 days.	1450	.14	.43	.....	.....
	Underburned Clay Brick					
H	Well laid, 1–3 portland cement mortar, 63 days.	1060	.27	.31	2870*	.37

*Average value based on 13 tests of 1–3 portland cement mortar cubes 60 days old.

data on the effect of the richness of the mortar in columns tested at the same age. For the columns with 1–3 portland cement mortar 67 days old, the ratio of the breaking load on the columns to the (average) ultimate load carried by a single brick tested flatwise in compression is 0.31; for 1–5 mortar, other conditions being the same, this value is 0.21. These values bear to each other nearly the same ratio as do the strengths of the mortars as shown by the results of tests of 6-in. cubes. This shows a decrease of 34 per cent due to the use of a leaner mortar. For the two columns built of 1–3 portland cement mortar which were tested at an age of 6 months, the value of this ratio is about 0.37, showing an increase of about 21 per cent in the strength of the column during the interval. No mortar cubes were tested at 6 months; but this increase of 21 per cent agrees well with the increase in strength of 1–3 portland cement mortar from 60 days to 6 months, as found in other laboratory tests.

The popping sounds referred to under "13, Phenomena of Tests of Brick Columns" have been accredited to the breaking of the brick in flexure due to the stresses introduced by the readjustment of the different parts following unequal shortening in different parts of the column at a given level, or to uneven bearing of the brick throughout their length, or to both. The practice of bricklayers in placing mortar at the ends of the bricks causes them to be more fully supported at the ends (or not to have a uniform bearing throughout their length). As the same thing is done in the course next above and the joints are broken, the effect is that any given brick has a greater load in the middle and its main support is at the ends. This is particularly true of the inner portion of each joint in a column of small section. It is therefore subject to flexure. After the bricks are broken and vertical cracks formed, this uneven distribution of the pressure and bearing has the effect of eccentricity of loading.

The load at which this popping was first observed is indicated by a cross (x) on the load-deformation diagrams given on pages 30 and 32. This load corresponds to a unit deformation of about 0.00049 for the columns built of 1–3 mortar and tested at 67 days. For similar columns tested at an age of 6 months the value of this unit deformation was somewhat higher, though not so well defined. With the leaner and weaker mortars the value of this deformation is from 0.0017 to 0.0135. For the clay-brick columns

(1–3 portland cement mortar) the deformation is 0.0023 and 0.0026. It seems probable that the popping noises are indications of incipient failure and that this load represents the maximum load which the column would continue to carry even under the conditions present during the test. By reference to the load-deformation diagrams it will be seen that this point coincides closely in nearly every case with the load at which a definite change in the nature of the curve occurs.

The two columns (No. 55 and 56) which were laid up hurriedly (poorly laid) give results which do not differ from those found in the columns built by the regulation methods as much as might have been expected. The average values show a decrease of strength of about 13 per cent due to this cause.

The ratio of the strength of the one column laid up with 1–3 Bricklayers' (natural) cement mortar to the strength of the shale brick columns built of 1–3 portland cement mortar is 0.52. The ratio of the strengths of the corresponding 6-in. cubes as given in Table 9, page 26, is 0.11.

No direct comparison of the strength of the lime-mortar columns with the strength of the mortar can be made (since no tests of the lime mortar were made), although it is evident that the low values observed are due to the low crushing strength of the mortar. From the action of these columns during the tests it seems evident that the lime mortar broke down at a load which was proportionally very much lower than that carried by the cement mortar. From the early signs of distress exhibited by these columns it seems doubtful if they would continue to carry a load greater than about one-third the maximum load given in the test, while for the shale brick columns built with portand cement mortar, the load at which the first signs of distress were observed is from 50 per cent to 75 per cent of the maximum load carried.

For the two columns built of underburned clay brick and 1–3 portland cement mortar, the average load carried was 1060 lb. per sq. in. The ratio of this load to the load taken by the shale brick columns tested at the same age is 0.31. The ratio of the crushing strength of the clay brick to that of the shale brick is 0.37.

It is evident that the strength of any brick or block structure is influenced greatly by the quality of the mortar used. These comparisons indicate that for the columns tested the strength of

columns built from the same brick is closely proportional to the strength of the mortar used in their construction, and the strength of columns built of different brick using the same mortar is closely proportional to the crushing strength of the brick. Of course, these conclusions may not be expected to hold for all combinations of brick and mortar. A mortar might be used of such low crushing strength that the strength of the poorest brick would be great in comparison.

These tests indicate that the crushing strength of the brick is as important a factor in the strength of a structure of this kind as the quality of the mortar used. The most economical structure would seem to result from using a mortar comparable in strength with the brick. Such considerations are generally unnecessary except in design of columns, piers, etc., which are to sustain excessive unit loads.

If, as pointed out in a preceding paragraph, the load at which the column first shows signs of distress by popping sounds represents the maximum permanent load which the column would carry, then this load is the one on which the factor of safety should be based, and not the load carried momentarily before failure.

The effect of eccentric loading and repeated loading on brick columns will be discussed in detail in succeeding paragraphs.

15. *Load-deformation Diagrams for Brick Columns.*—In Fig. 5 and 6, the diagrams show the relation of longitudinal deformation to the applied load for the brick columns tested. The make-up and loading of each column may be found by reference to Table 7. The deformation and load at which the first popping noises were heard are indicated on the diagrams by a cross (x). The curves represent the average of the deformations observed on two opposite faces of the columns. They were plotted from the extensometer measurements and the applied loads. The deformations were measured over a gauged length of about 100 inches.

The curves show the rate at which the column is shortening at any particular load. It is seen that there is not a direct proportionality between the applied load and the resulting deformation, though there is an approach to this for the lower loads. If a straight line be drawn through the points marking the earlier loads, (generally speaking, tangent to the curve), it will represent what would be the modulus of elasticity of the column if it

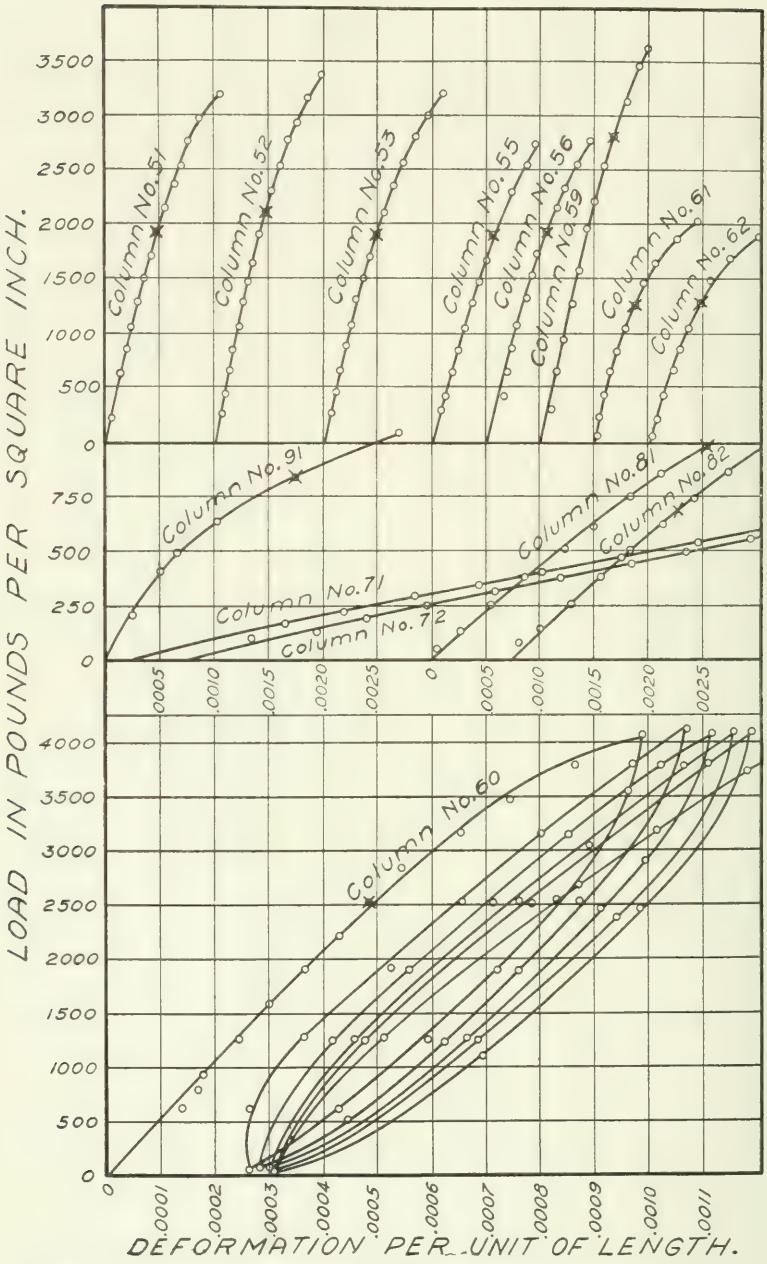


FIG. 5. LOAD-DEFORMATION DIAGRAMS FOR CONCENTRICALLY LOADED BRICK COLUMNS.

continued to compress at the same rate for all loads as it did for the earlier loads. The modulus of elasticity, found from the line so drawn, is termed the "initial modulus of elasticity". The value of this function is given for the brick columns in Table 8, page 25.

The load-deformation curves for the two columns (No. 57 and 58) which were loaded eccentrically are given in Fig. 6, page 32. The maximum deformation is at one face (the face nearer the point of application of the load), and the minimum deformation is at the opposite face. Between the two faces, on the assumption of a plane section before loading remaining a plane section after the load is applied, the deformation will vary uniformly between the values given on the diagram for the corresponding loads on the two faces. The interpretation of the variation of the stresses across the section is given in another place.

16. *Effect of Repeated Loads on Brick Columns.*—One brick column (No. 60, shale brick, 1-3 portland cement mortar, 6 months old) took the full load of the testing machine (651 000 lb., 4110 lb. per sq. in.) without failure. This load was released and reapplied seven times. Upon the seventh application of the load, before the instruments could be read, the column failed with little warning. The failure was complete, as described above for this class of columns. This column is exceptional in the load carried without failure. The corresponding column (No. 59) failed under a single application of a load of 601 000 lb. The stress-deformation diagram for Column No. 60 is given in Fig. 5, page 30. The initial modulus of elasticity for the first application of load is very high (5 400 000 lb. per sq. in.). The value of this function for the succeeding applications is somewhat reduced, being about 4 500 000 lb. per sq. in. The curves for the first application of load are similar to those of other columns of this class. Throughout the repetition, the observed deformations on two opposite faces of the column were nearly the same, showing the absence of bending. The diagram shows the gradual increase in the total deformation under succeeding applications of the load. Upon release of the load there is a "set" which is nearly equal to the increase in total deformation due to the previous application of the load. The curve for decreasing load is seen to be almost the reverse of that for increasing loads (concave upward). This phenomenon has been termed the "inertia of strain", i. e., the lag of

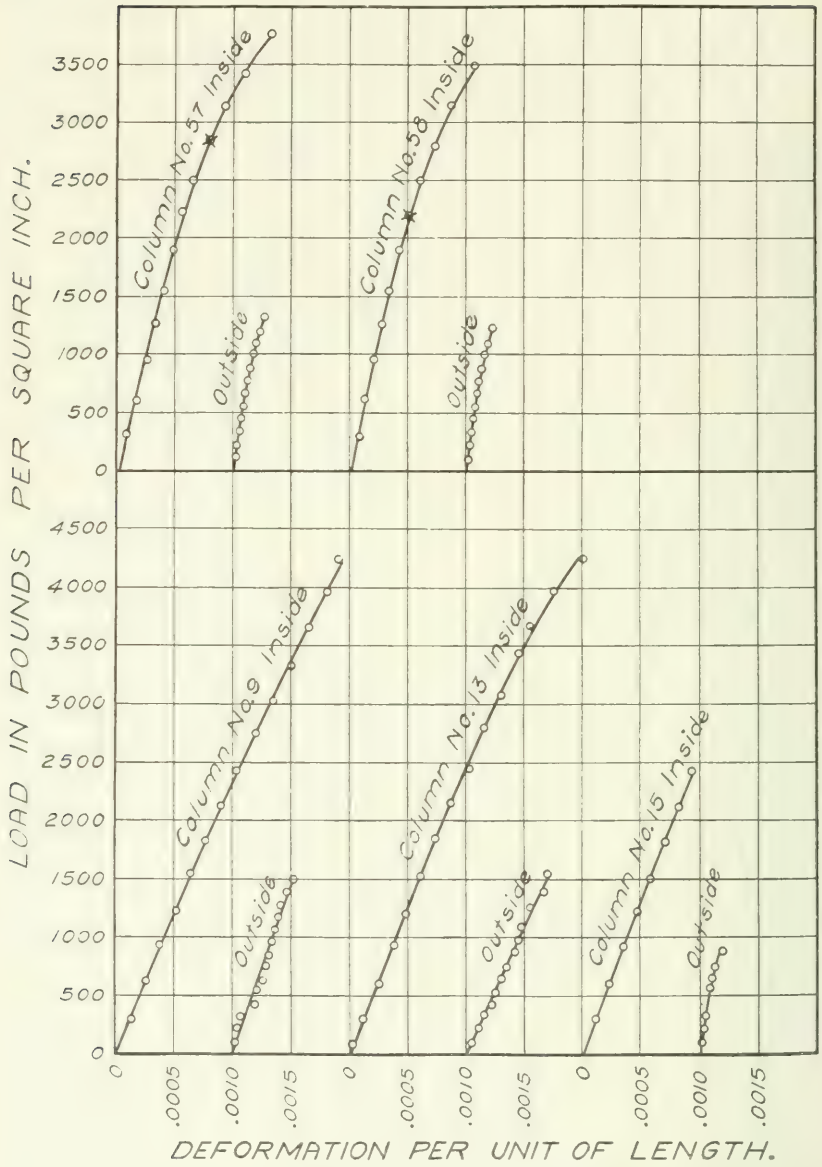


FIG. 6. LOAD-DEFORMATION DIAGRAMS FOR ECCENTRICALLY LOADED BRICK AND TERRA COTTA BLOCK COLUMNS.

the deformation behind the load. This effect is particularly noticeable at the beginnings and ends of the curves where there is a reversal in the character of the increments of load. It is probable that this column would have taken a load of about 700 000 lb. (4370 lb. per sq. in.) upon a single application to failure. This conclusion is based upon the amount of deformation which probably would have been required to produce failure. It is plain that the release and reapplication of a load produces failure at a lower load than would be necessary at a single application. It seems possible that the indefinite reapplication of the load which produced the first popping sounds mentioned above might finally produce failure. This load corresponds to a load 58 per cent of the load that the column probably would have carried upon a single application.

17. *Effect of Eccentric Loading of Brick Columns.*—In the two brick columns (No. 57 and 58) which were loaded eccentrically, the load was applied at either end of the column along a line 1 in. off the middle of the section, as described under "11. Methods of Testing". An amount of eccentricity of the load was chosen so that none but compressive stresses might exist in the column. For a rectangular column having the property of proportionality of stress and deformation, in order that tensile stress be not developed, the point of application of the load should fall within the middle third of the section. For such a material as brick masonry it is evident that, as the deformation will not be proportional to the stress at the higher loads, the distribution of the stress across the section may differ somewhat from that assumed in the ordinary analysis. However, this analysis will be used as a basis of comparison with the stresses determined from the observed deformations.

The formula for maximum and minimum stress in a rectangular column subjected to eccentric loading, based upon proportionality of stress and deformation, and neglecting the further eccentricity due to the bending of the column, is

$$f = \frac{P}{A}\left(1 + \frac{6\epsilon}{d}\right)$$

where f is the unit-stress at the inner or outer face of the column, respectively, P is the total load on the column, A is the area of the section of the column, ϵ is the eccentricity of loading or distance from the line of application of the load to the center line of

the section, and d is the dimension of the column perpendicular to the line along which the load is applied. For a value of ϵ of 1 inch and a side d of 12½ in., this formula becomes

$$f = \frac{P}{A}(1 + 0.48).$$

In other words, the stress at the inner face is 48 per cent greater, and at the outer face it is 48 per cent less, than the average stress over the section of the column. The values of the minimum stress and the maximum stress corresponding to several average stresses are given in Table 10, page 34.

To determine the minimum stress and the maximum stress on the column from the deformations observed at any load, use may be made of the stress-deformation diagram of a column centrally loaded. The stress corresponding to the deformation of the eccentrically loaded column may in this way be taken direct from the diagram for the centrally loaded column. Three columns (No. 51, 52, and 53) are comparable to the two eccentrically loaded columns in materials and fabrication, and their load-deformation diagrams may be considered to give properties representative of the latter two columns. For comparison, the deformations at the inner and outer faces of No. 57 and 58 were taken from their diagrams for several loads, and the unit-stresses correspond-

TABLE 10.

COMPARISON OF STRESSES IN ECCENTRICALLY LOADED BRICK COLUMNS

Stresses are given in pounds per square inch.

	Average Unit Stress over Section of Column	Stress at Inner Face of Column		Stress at Outer Face of Column	
		From Deformations	From Formula	From Deformations	From Formula
Column No. 57	500	840	740	245	260
	1000	1590	1480	430	520
	1500	2290	2220	600	780
	2000	2900	2960	780	1040
	2500	3760	3700	1065	1300
Column No. 58	500	702	740	272	260
	1000	1340	1480	492	520
	1500	2105	2220	675	780
	2000	2840	2960	845	1040
	2500	3570	3700	1127	1300

ing to these were found by the use of the load-deformation diagram of each of the three centrally loaded columns named. The average of the three values of the stresses so determined for each case is given in Table 10, page 34, in the columns headed "From Deformations". It will be seen that the values determined from the deformations agree fairly well with the results of the formula, considering the indefiniteness in the amount of eccentricity and the lack of precision in measuring the deformations.

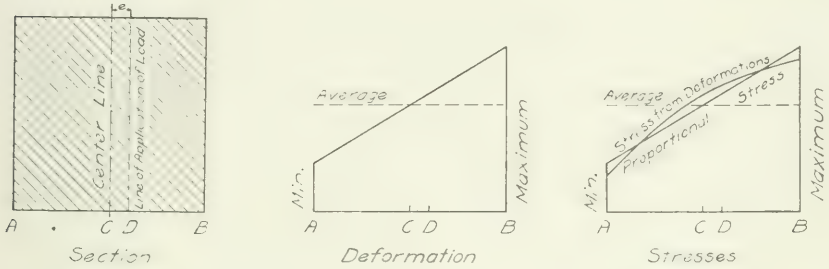


FIG. 7. DISTRIBUTION OF DEFORMATION AND STRESS IN AN ECCENTRICALLY LOADED COLUMN.

The stress at the inner face for the load which produced failure, calculated by the formula given above, is 4030 lb. per sq. in. for No. 57, and 4260 lb. per sq. in. for No. 58. This is considerably greater than the stress on the three centrally loaded columns at failure (average, 3365 lb. per sq. in.). The material under maximum stress is evidently restrained and aided by that near it. It is worth while, also, to call attention to the effect of eccentric loading, since for an eccentricity of 1 inch the columns failed at an average load of 2800 lb. per sq. in., while the centrally loaded ones carried 3365 lb. per sq. in., indicating a loss of 17 per cent in carrying capacity due to the eccentric loading. This result, obtained with a small eccentricity, emphasizes the need of providing for such stresses, or of designing the structure to avoid them so far as possible.

It is interesting to note that although the deformation at the inner face was 67 per cent more than the average, and that at the outer face as much less, the stresses found at the two faces are only about 48 per cent more and less than the average. It may also be of interest to compare the stresses at different points across the column.

In Fig. 7, page 35, the middle diagram shows the distribution of the deformation across the column, assumed to vary uniformly. The diagram on the right gives the distribution of stresses. The straight line in the latter stress-deformation diagram represents the assumptions of the ordinary analysis. The curved line gives the stresses determined from the deformations by the method used in finding the stresses at the outer and inner faces given in Table 10, page 34.

B. TERRA COTTA BLOCK COLUMN TESTS.

18. *Terra Cotta Block Column Tests.*—Sixteen terra cotta block columns were tested. Table 11, page 39, gives data on the dimensions, make-up, and method of loading the columns. The columns were built and tested in two lots, an interval of about one year separating the times of making the tests. The two lots of columns were built of blocks which came in different shipments. The cement used was the same brand in both years, though the lots were different. The terra cotta block columns were generally made in sets of two. Each set was constructed and loaded similarly. Three of the columns were laid up hurriedly (poorly laid); the remainder were built with the usual care given to such work, as described on page 17. The number of variables in these tests was smaller than was the case in the tests of brick columns, but the variation in materials makes comparisons of results more difficult. The load was applied to the columns in different ways. Generally the load was applied continously to failure. In two cases (No. 7 and No. 8) the maximum load that could be applied with the machine was repeated several times without loading the columns to failure. No. 7 was loaded eccentrically also but not enough load was applied in this way to cause failure. These two columns were removed from the testing machine and have been set aside for future tests. Three columns were loaded eccentrically to failure. Two columns (No. 6 and No. 7) had an increasing load applied and released in succession. This method of loading will be understood by reference to the stress-deformation diagrams for these columns, Fig. 10, page 46.

19. *Phenomena of Tests of Terra Cotta Block Columns.*—The terra cotta block column tests resembled the tests of brick columns in many respects. In the following paragraphs it will fre-

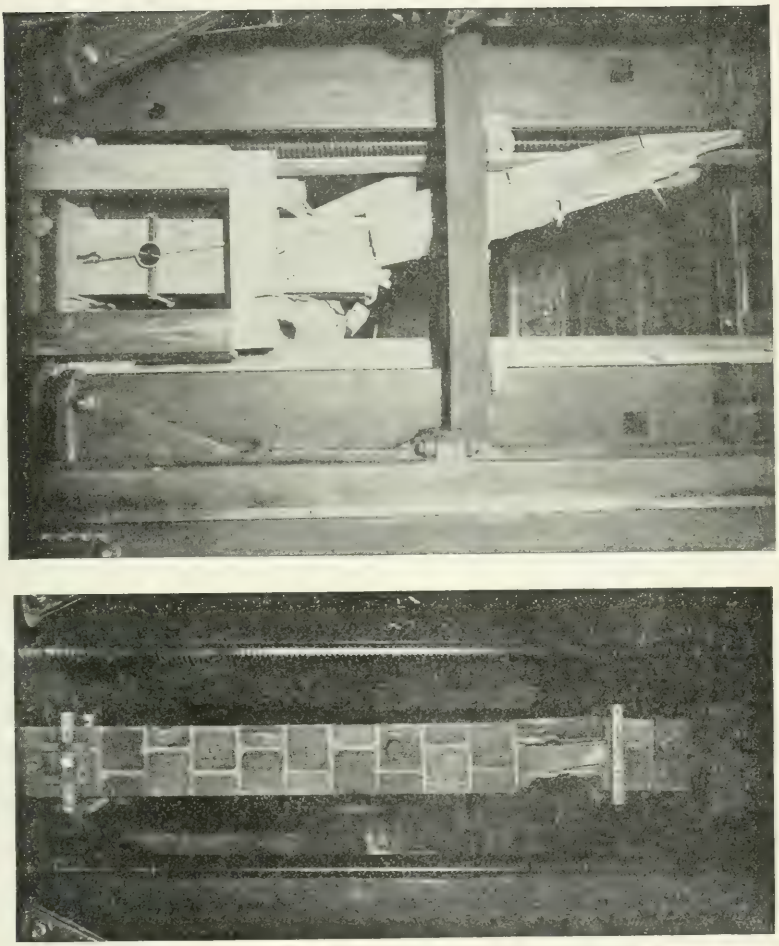


FIG. 8. VIEWS SHOWING MANNER OF FAILURE OF TERRA COTTA BLOCK COLUMNS NO. 4 AND NO. 18.

quently be convenient to refer to the parallel description of the tests of brick columns to avoid repetition.

Generally the terra cotta block columns gave no sign of distress until a load near the maximum was applied, when cracking noises, similar to those described for the brick columns, were heard. This was soon followed by the formation of longitudinal cracks through the vertical joints or the spalling of the horizontal mortar joints at the corners of the column, either of which was immediately followed by sudden collapse of the column. The failures of these columns were even more violent than those described for the brick columns. The notes for test of Column No. 14 state that there were 18 whole blocks found in the debris after the collapse of the column: over 70 per cent of the blocks were broken. The percentage of breakage was not so great in the columns of smaller section, since failure in these was more local. The smaller columns showed more variation in the manner of failure than the large ones. The characteristic form of failure for the 12½ x 12½ in. columns was a sudden total collapse immediately following the appearance of longitudinal cracks through the vertical joints near the middle. Failure sometimes occurred with no warning except the continued shortening of the column under the increasing load. The photographs (Fig. [illegible] page 37) show failures of some of these columns which are characteristic.

The columns loaded eccentrically failed by splitting from end to end along the vertical mortar joint which was parallel to and nearer the loading bar. In the test of No. 15 under eccentric load, after having been loaded centrally, failure came prematurely by the breaking of the lower bearing block due to the excessive cross-breaking stress arising from this method of applying the load to the column. The load at failure can not be considered to be the strength of this column even under eccentric loading.

The fractured surfaces of No. 17, (built of apparently inferior blocks) showed many light-colored interiors. This appearance confirms the judgment that was exercised in selecting these as underburned blocks.

20. *Strength of Terra Cotta Block Columns.*—The results of the test of terra cotta block columns are given in Table 12, page 40. A summary of average values from the tests is given in Table 13, page 42. The variation in materials and the method of applying the load, together with the inability to load some of

TABLE 11.

Data of Terra Cotta Block Columns.

Col. No.	Characteristics of Column and Loading	Kind of Mortar	Length ft.-in.	Nominal Section in. x in.	Gross Area sq. in.	Number of Courses	Average Thickness of Joints inches
			Columns tested January, 1907.				
1	Well laid, loaded concentrically	1-2 portland	12-6	8½ x 8½	72.8	18	.33
2	do.	do.	12-6¼	8½ x 8½	72.8	18	.35
3	do.	do.	11-11	8½ x 13	110.5	17	.41
4	do.	do.	11 10½	8½ x 13	112.1	17	.38
5	do.	do.	12-7¼	13 x 13	170.5	18	.41
6	do.	do.	12-7	13 x 13	169.0	18	.39
7	Loaded concentrically and eccentrically	do.	12-7¾	17½ x 17½	307.3	18	.42
			Columns tested January, 1908.				
8	Well laid, loaded concentrically	1-3 portland	9-9	12½ x 12½	162.6	14	.34
15*	do.	do.	9-9	12½ x 12½	162.6	14	.34
9	Well laid, loaded eccentrically	do.	9-9	12½ x 12½	162.6	14	.36
14	Poorly laid, loaded concentrically	do.	9-9	12½ x 12½	161.3	14	.36
13	Poorly laid, loaded eccentrically	do.	9-9	12½ x 12½	161.3	14	.36
12	Poorly laid, loaded concentrically	do.	9-9	12½ x 12½	162.5	14	.36
17	Well laid with inferior block, concentric load	do.	9-9½	12½ x 12½	163.8	14	.39
18	Well laid, loaded concentrically	1-5 portland	9-10	12½ x 12½	164.3	14	.43
19	do.	do.	9-10	12½ x 12½	161.3	14	.43

*Later loaded eccentrically to failure.

TABLE 12.

DATA OF TESTS OF TERRA COTTA BLOCK COLUMNS

Col. No.	Maximum Total Load pounds	Unit Load lb. per sq. in.	Age at Test days	Initial Modulus of Elasticity lb. per sq. in.	Maximum Unit Shortening
Columns tested January, 1907.					
1	219 000	3030	68	2 170 000	.00160
2	200 000	2740	66	1 910 000	.00153
3	300 000	2700	67	No readings taken	
4	388 100	3440	68	2 150 000	.00175
5	543 400	3200	65	2 040 000	.00200
6	458 000	2710	68	2 700 000	.00142
7	607 700+	1980+	69	2 200 000	
Columns tested January, 1908					
8	606 000+	3730+	68	2 780 000	.00154
15	616 000+	3790+	63	2 750 000	*.00170
9	565 000	3470	64	2 330 000	
14	538 000	3330	63	2 860 000	.00134
13	501 000	3110	67	2 500 000	
12	534 000	3280	69	3 200 000	.00134
17	500 000	3050	68	2 300 000	.00196
18	526 000	3200	66	2 680 000	.00150
19	564 000	3500	64	2 700 000	.00148

*Based upon the deformation produced by the first application of a central load.

The loads in table followed by a plus (+) sign are not the ultimate loads of the columns, but are the maximum loads that could be applied with the testing machine used.

the columns to failure as planned, make comparisons of results uncertain in some cases.

It will be seen by comparison of the results of the tests made January, 1907, that there is no appreciable difference in the strength of the columns due to a variation in the size of the section. It was anticipated that column No. 7, $17\frac{1}{2}$ x $17\frac{1}{2}$ in. in section, could not be loaded to failure, but it is of interest to know

NOTES ON MANNER OF FAILURE OF TERRA COTTA BLOCK COLUMNS

Col. No.	Manner of Failure
	Columns tested January, 1907
1	Three top courses shattered.
2	Failure followed appearance of crack in 7th and 8th courses from bottom.
3	Compression failure at middle on west side.
4	Crushed at top. Broke in middle.
5	Cracked at top.
6	Horizontal joint spalled in 12th course. Top half of column collapsed.
7	No sign of failure at full capacity of machine. Set aside for future test.
	Columns tested January, 1908
8	Did not fail. Set aside for future test.
15	Did not fail. Later loaded eccentrically.
9	Split end to end along joint.
14	Shattered suddenly, following appearance of vertical cracks near middle.
13	Split end to end along joint.
12	Shattered suddenly, following appearance of vertical cracks near middle.
17	Completely shattered, following formation of vertical cracks near middle.
18	do.
19	do.

that the elastic properties of a column of this size (as shown by the load-deformation curve) are similar to those of smaller columns. The ratio of the maximum loads carried by the 1907 columns which were tested to failure to the crushing strength of individual blocks is about 0.86.

No tests were made of the mortar used in Columns No. 1 to 7, inclusive. The tensile tests of 1–3 mortars, given in Table 5, page 15, indicate that the two lots of Universal portland cement were similar. From this we may conclude that the rich mortar used in the 1907 columns was proportionally stronger than the

mortar used in the 1908 columns. The great difference in the strengths of the columns built from the two shipments of blocks seems to be due to the form of the blocks themselves. As was pointed out in a previous paragraph, the blocks in the first lot were cut with their ends (bearing faces) concave in the form of a

TABLE 13.

SUMMARY OF TESTS OF TERRA COTTA BLOCK COLUMNS

Average Values

Ref.	Characteristics of Columns	Numbers of Columns Used in Average	Average Maximum Unit Load lb. per sq. in.	Ratio of Strength of Column to Strength of Block (Gross area)	Crushing Strength of 6-in. Mortar Cubes lb. per sq. in.	Ratio of Strength of Column to Strength of Cubes	Ratio of Strength to Estimated Strength of "E"
	Columns tested January, 1907						
	1–2 portland cement mortar. All well laid and centrally loaded.						
A	8½ x 8½ in.	1, 2	2885	.83			
B	8½ x 13 in.	3, 4	3070	.89			
C	13 x 13 in.	5, 6	2955	.85			
D	17½ x 17½ in.	7,	1980+	....			
	Columns tested January, 1908						
	12½ x 12½ in., 1–3 portland cement mortar, well laid unless noted.						
E	Central load	8, 15,	3790+ 4300*	.74+ .83*	3400	1.12+ 1.26*	..... 1.00*
F	Eccentric load	9,	3470	.65	3090	1.12	.81*
G	Poorly laid, central load	12,	3305	.64	3130	1.05	.76
H	Poorly laid, eccentric load	13,	3110	.60	3025	1.06	.75
I	Inferior blocks, central load	17,	3050	.59	3370	.88	.71
K	1–5 mortar, central load	18, 19,	3350	.65	.....	.....	.78

*Estimated.

The average age of the columns when tested was 67 days.

cylindrical surface. The variation from a plane was in some cases as much as $\frac{3}{16}$ in. The blocks from the second shipment had end faces more nearly plane. The variation in strength of these blocks as found from the tests of individual blocks as well as in the column tests seems to be due to the form of the bearing faces. Other comparisons of the two lots of blocks show them to be similar, both as to material and manufacture. The blocks from which the 1907 columns were made, failed in the compression test at a lower load apparently on account of the non-uniform distribution of the load over the section. The blocks crushed first at the high points of the ends, and complete rupture soon followed. Fig. 9, page 43, shows the conditions which seem to

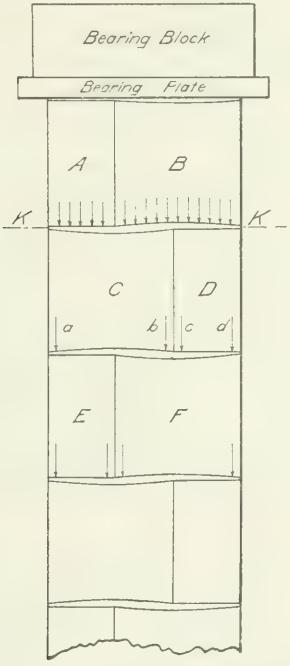


FIG. 9. DISTRIBUTION OF PRESSURE IN A COLUMN BUILT OF BLOCKS WITH CURVED ENDS.

exist in a column built of the 1907 blocks. The amount of curvature in the faces of the blocks has been somewhat exaggerated in this sketch. Some course of blocks near the top as A and B, may be assumed to receive a uniformly distributed load and

transmit it as such to the joint below. The curvature of the ends of the blocks gives a variable thickness to the joint below, and on account of the greater amount of yielding in the thicker part of the joint, the block D will deliver its load somewhat concentrated at d. Similarly, the pressure transmitted downward by C will be greater at a than at b or at points between. This change in the distribution of the pressure will produce a horizontal force tending to open the vertical cracks. Likewise any concentration of the load on any block F near its middle accompanied by a concentration of the supporting pressure below at points near its ends will tend to produce cross breaking along a vertical line. It is clear that the conditions described would become intensified as the load is transmitted to each lower course, until the middle of the column is reached. This explanation of the distribution of the stresses may account for some of the phenomena of these tests. If the blocks safely resisted the stresses tending to split them, there remained the danger of crushing the mortar at the corners due to excessive pressure existing there. The columns built from 1907 blocks failed by splitting the blocks through a longitudinal joint or by crushing the mortar at the corners. Under ordinary conditions the mortar should carry a larger load than came on these columns.

None of the columns well laid, using 1908 blocks and 1–3 mortar, failed by applying a central load up to the capacity of the testing machine. It is estimated from the character of the load-deformation curves for Columns No. 8 and No. 15 and from the strength of the blocks, that these columns would have required a load of about 4300 lb. per sq. in. to cause failure. The strength of similar columns under eccentric load also points to the same conclusion.

Column No. 12, which was laid up hurriedly, gave results relatively much lower than have been found in the tests of brick columns under similar conditions. Similar columns (No. 13 and No. 14) which were loaded eccentrically gave values nearly as large as No. 12. The strengths of the poorly-laid columns under eccentric load seem to be more nearly representative than the centrally loaded one.

The column (No. 17) laid up with inferior blocks gave values about 71 per cent of the estimated strength of the similar columns built of better blocks.

The two columns (No. 18 and No. 19) built of 1–5 portland cement mortar gave results about 22 per cent lower than the estimated values for the 1–3 mortar columns.

21. *Load-deformation Diagrams for Terra Cotta Block Columns.*— Longitudinal deformations were measured on all the terra cotta block columns except No. 3. The load-deformation curves have been plotted for each column in the same manner as was described for the brick columns. In some cases where the same column

TABLE 14.

COMPARISON OF STRESSES IN ECCENTRICALLY LOADED TERRA COTTA BLOCK COLUMNS.

Stresses are given in pounds per square inch.

	Average Unit Stress over Section of Column	Stresses at Inner Face of Column		Stresses at Outer Face of Column	
		From Deformations	From Formula	From Deformations	From Formula
Column No. 7	500	1125	1010	200	—12
	1000	1940	2025	315	—25
Column No. 9	500	718	735	252	265
	1000	1340	1470	487	530
	1500	1930	2210	603	795
	2000	2530	2940	.....	1060
Column No. 13	500	900	735	350	265
	1000	1700	1470	680	530
	1500	2350	2210	1060	795
	2000	2950	2940	1400	1060
	2500	.....	3680	1700	1225
Column No. 15	500	750	735	100	265
	1000	1500	1470	250	530
	1500	2500	2210	400	795

Column No. 7. $d = 17.55$ in. $e = 3$ in.

Stresses from deformations are those corresponding to the first application of a previous central load on the same column. Stresses from formula are about 100% more or less than the average stress across the section.

Column No. 9. $d = 12.75$ in. $e = 1$ in.

Stresses from deformations were computed from deformations of Columns No. 8 and 15, for first application of central load. Stresses from formula are 47% more or less than the average.

Column No. 13. $d = 12.75$ in. $e = 1$ in.

Stresses from deformations were computed from deformations of Columns No. 12 and 14.

Column No. 15. $d = 12.75$ in. $e = 1$ in.

Stresses from deformations were computed from deformations for first application of central load on Columns No. 8 and 15.

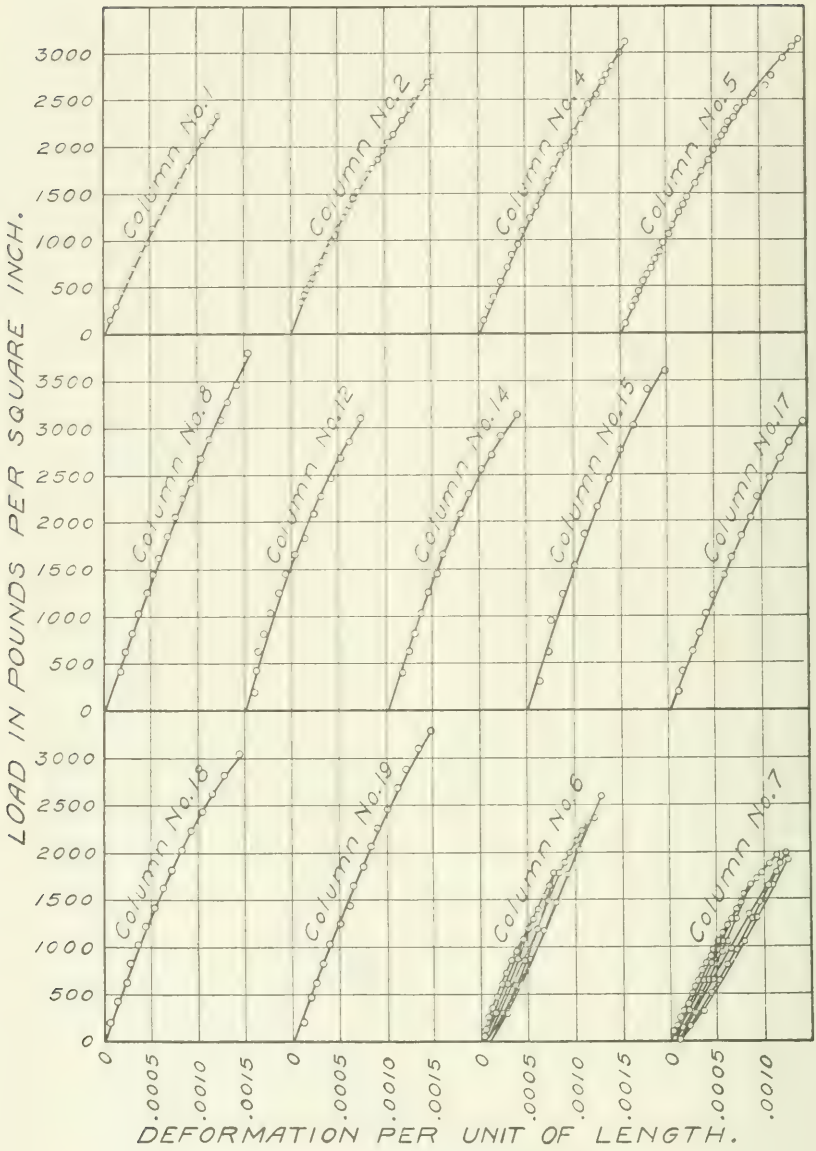


FIG. 10. LOAD-DEFORMATION DIAGRAMS FOR CONCENTRICALLY LOADED TERRA COTTA BLOCK COLUMNS.

was loaded in different ways two diagrams are given. In cases of this kind the form of the curve itself is sufficient to indicate the nature of loading. Load-deformation diagrams for the terra cotta block columns are given in Fig. 6, page 32, and Fig 10, page 46.

22. *Effect of Eccentric Loading of Terra Cotta Block Columns.*— Four of the terra cotta block columns were loaded eccentrically. The amount of the eccentricity was 1 in. in each case, except Column No. 7, where $e = 3$ in. Table 14 page 45, gives a comparison of the stresses across the section of the column as determined by two methods of computation. This table was prepared in the way described for Table 10 for the brick columns. The notes following the table give in detail the origin of the values for each column.

Column No. 7 was not loaded to failure. No. 15 failed prematurely under eccentric load due to the breaking of the bearing block on top of the column and the resulting concentration of the load. This accounts for the smaller number of stresses given in the comparisons for these columns in Table 14.

The terra cotta block columns under eccentric load exhibited phenomena very similar to those observed for the brick columns. The stresses computed from deformations agree fairly closely in each case with those derived from theoretical considerations based on the measured amount of the eccentricity of the load.

23. *Effect of Repeated Loads on Terra Cotta Block Columns.*— On columns No. 7, 8, and 15, the maximum load that could be applied with the testing machine was applied and released a number of times. In no case was failure produced by a re-application of the load, although it seems evident from the increasing amount of deformation produced by the maximum load and the gradually increasing permanent set upon release of the load on Column No. 15 that only a few more applications of this load would have been necessary to produce failure. The maximum deformation produced in Column No. 15 (load-deformation diagram for repeated load not given) by the re-application of the load was 0.00168. The great increase of deformation due to the re-application of these high loads is convincing evidence of the weakening effect of such loading. This column was later loaded eccentrically, but its premature failure due to the breaking of the top bearing block prevented any independent estimate of its original

strength. The similarity in the elastic properties of Columns No. 8 and No. 15 indicates that they probably would have carried nearly the same loads at failure. The comparatively low unit loads which were reapplied to Column No. 7 produced a correspondingly increased total deformation and appreciable set upon release of the load.

C. COMPARISON OF RESULTS.

24. *Summary.*—Both brick columns and terra cotta block columns gave high strengths in all cases where strong mortar and care in building were used. For central loading, the strength of the brick columns ranged from 3220 to 4110 lb. per sq. in., and the strength of the terra cotta block columns from 2700 to 3790 lb. per sq. in., the columns having the highest resistance not failing at the full capacity of the machine. The effect of the strength of the mortar is apparent in the carrying capacity developed in the columns: lower loads were found in columns built with 1–5 portland cement mortar than in those with 1–3 portland cement mortar, still lower loads in those with 1–3 natural cement mortar, and still lower loads in those having 1–2 lime mortar. The effect of the quality of the brick is shown in the columns made with inferior brick, which carried only 31 per cent as much as columns built with the better grade of brick. In the case of the terra cotta columns, the blocks which were culled out as somewhat inferior gave a column strength perhaps 30 per cent less than the columns built with superior blocks. The effect of the attempt to represent hurried or careless workmanship in two brick columns and in three terra cotta block columns was a loss in strength of about 15 per cent and 25 per cent, respectively.

The ratio of the strength of the columns to the compressive strength of the individual brick and block is of interest. In the well-built brick columns loaded centrally, the ratio of strength of column to compressive strength of individual brick ranged from 0.31 to 0.37, and in the underburned clay brick column the ratio was 0.27. In the terra cotta block columns with central loading the ratio of strength of column to that of individual block was 0.74 for the incompleted test and 0.83, 0.85, and 0.89 for the others. If, as seems to be the case, the strength of the brick or block to resist cross-breaking is an element in determining the

strength of the built-up column, a deeper or thicker brick would give higher column strength. It is possible that this partially accounts for the fact that the ratio is found to be higher for terra cotta block columns than for brick columns. The tests suggest that the ability of individual pieces to resist transverse strength is an important element in the strength of the completed column. This suggestion may have an important bearing on the advantageous size of the component blocks which may be used in a compression piece where high strength is desired.

The strength of the column is greater than the strength of the mortar cubes in both brick and terra cotta block columns, excepting only the soft brick columns which had brick of low compressive strength. It is evident that the strength of individual brick or blocks and the strength of the mortar both enter into the resistance of the column. The relative effect of the two depends upon the character of the material. It is evident, however, that the better the individual piece the more important it is to have a mortar of high resisting strength.

The results obtained in applying the load eccentrically were found to agree very well with those obtained from ordinary analysis. When the amount of eccentricity in the application of the load is known or may be estimated closely, the ability of the column to resist this action may be calculated quite closely. It is apparent from the results that the calculated resisting stress in the column on the side of maximum compression is higher than that which causes failure in centrally loaded columns. The higher stress developed with eccentric loading is probably due to the influence of the restraint of the less stressed interior portion. The tests made by applying and releasing a single load a number of times gave failures at loads below those which produced failure in similar columns at a single application of the load. The phenomenon is common in materials of the nature of brick and terra cotta.

It is apparent that the quality of workmanship in laying up such columns has an important bearing upon the resisting strength. The work of building columns, however, is not difficult and requires only ordinary care. Full joints and an even bearing are important, and the ordinary workman ought to be able to construct columns of high strength. In the tests made on columns

intended to represent poor or careless workmanship, the decrease in strength was not as much as anticipated. However, it must be understood that careful and trustworthy work is essential and that a few poor joints will materially reduce the strength of the structure. Wherever good material and good workmanship are insured the strength of masonry of this kind may be utilized with advantage.

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